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Montana. Highway Dept. Materials Division
Beneficiation of aggregates

Report on
BENEFICIATION OF AGGREGATES
for
Upgrading of Substandard Native Materials
in Eastern Montana

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BENEFICIATION OF AGGREGATES

Research studies on the possibilities of utilization and upgrading substandard native materials in Eastern Montana.

Final Report

1966

Research Project HPR 1(3): Item R-8

No. 7904

by

MONTANA HIGHWAY DEPARTMENT

Materials Division

in cooperation with

U. S. Department of Commerce

BUREAU OF PUBLIC ROADS

TABLE OF CONTENTS

Preface ii

Introduction 1

Geology 1

Soils 3

Climate 4

Topography 4

Construction Materials 4

Test Methods 7

Base Course Investigations:

  Gravel 9

  Quarry Materials 10

  Sand 10

  Portland Cement Stabilization 11

  Sand-Asphalt Stabilization 11

Surfacing Courses 14

Baked Shale Surfacing 16

Subgrade 17

Conclusions:

  Base Course 18

  Surfacing 19

Design Considerations 19

Appendix

(Figures numbered 1 through 15)

PREFACE

Immediately after World War II, many highway projects were programmed and scheduled for construction and reconstruction. During the design stage for these projects, it soon became apparent that portions of eastern Montana had, or shortly will have, inadequate quantities of good quality gravel for use in the construction of highways. In order to counteract the very expensive construction which could result from the extremely long haul of good quality materials, it was necessary to find some means to upgrade the substandard materials prevalent in this area. Basically, this study was undertaken to investigate the possibility of using some of the widely distributed sand, clayey sand and substandard gravel in place of the good materials that had previously been used in base and surfacing courses. The quality of the substandard materials will have to be improved by the addition of some agent, such as portland cement, asphalt, chemicals or admixtures of clay.

Fine non-plastic sands have been used for base course (unstabilized) and asphaltic surfacing on several experimental projects and as maintenance betterments. Although the experimental projects were generally disappointing, they did indicate several possibilities for obtaining more satisfactory results. The maintenance work also offered some pertinent information on the use of substandard materials. Observations over a period of years indicated that certain combinations of sand and clay gave adequate subgrade stability for some of the light traffic secondary roads. The extent to which this stability is dependent upon favorable drainage and moisture is not clear, but warrants investigation.

Several states, notably Iowa and Nebraska, have experimented with sand and substandard gravel for both base course and asphaltic surfacing. It is understood the test sections that were constructed on the basis of their research findings have rendered good service under light to medium traffic

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conditions. Hence, this project was initiated to study means and various materials necessary to upgrade the substandard materials in eastern Montana areas. The original title, "Upgrading of Substandard Materials in Eastern Montana", was revised and shortened to "Beneficiation of Aggregates" so as to be able to include other substandard materials in different parts of the State when such materials were encountered.

UPGRADING OF SUBSTANDARD MATERIALS IN EASTERN MONTANA

INTRODUCTION

This project originated as a research study of a typical area in eastern Montana where gravel is poorly distributed. The major purpose of this study was to investigate the possibility of upgrading some of the substandard materials for use as roadbuilding aggregate. Work was started on September 1962. This report covers field examination of the area, general sampling to determine limits of acceptable or potentially acceptable materials, and laboratory investigation.

The area under study is in the triangle formed by the convergence of the Missouri and Yellowstone Rivers toward the east, and the Musselshell River in the west. It includes all of Garfield County, the western part of McCone County and the northern part of Rosebud County (figure 1). Figure 2 shows the major highways, towns, and drainages in the area.

The work was aimed at defining the problems and examining some of the more conventional approaches. The area was examined to determine the continuity and extent of the various materials considered in their geological setting. Sampling was guided by (1) exposures of geologic formations with concentration on obtaining a good geographic distribution; (2) delineating quality of materials; and (3) natural conditions providing clues to the stability or durability of the materials.

GEOLOGY

The geology of this area as shown in figure 2 is relatively simple. In the southeastern part of the area Cretaceous formations are

extensively exposed along such major structural features as the Porcupine Dome, the Cat Creek anticline, and the Sumatra anticline and syncline. The remainder of the area is covered by Tertiary Fox Hills and Fort Union formations. In parts of Montana, the Fort Union is broken down into Tongue River sandstone member at the base. Lebo shale in the middle, and Tullock sandstone at the top. Although the shale member is extensively exposed east of Van Norman in the study area, the formation has not been sub-divided on the available geology maps.

The only igneous rock in the area is a steeply dipping, black fine-grained dike which causes a series of small buttes northwest of Jordan; the largest of these is called Smoky Butte. The dike is variable in width (maximum of about 70 feet) and is bordered by very altered sedimentary rocks. Only the core of the dike is hard and durable material.

In general, the various sedimentary formations are soft and weather rapidly, but throughout the area, thin beds of moderately hard sandstone form a caprock protecting the softer materials. It appears this hardening is at least partly a weathering phenomena and the hardness may be considerably less away from the outcrop. This possibility would have a decided effect on efforts to produce quantities of harder sandstone.

Another moderately hard material in the upper part of the Fort Union formation (Tongue River sandstone member) is baked shale and siltstone resulting from the burning of lignite beds exposed at the surface. This baked material, which is locally referred to as scoria, is widely distributed as small deposits in the eastern part of the area.

The intensity of the heat is somewhat indicated by the intensity of the color with yellow and deep red to black indicating the hottest zones. The shale has been most intensely altered by this process and in many cases it has been recrystallized or fused into a lightweight non-swelling material; shaly siltstone was also considerably affected but the sandstone shows little physical evidence of change except for coloration.

The northern part of this area was glaciated and glacio-fluvial or glacial deposits were formed in an area along the Musselshell River on the west, Dry Creek in the center, and Redwater River on the east. Glacial deposits or stream concentration of glacial materials are the sources of almost all of the gravel deposits in eastern Montana; thus, the limited extent of glaciation, the absence of major streams except on the periphery of the area, and the absence of suitable quarry materials have created a problem which is the reason for this study.

SOILS

With the exception of the small areas covered by glacio-fluvial deposits, the soils throughout the area are derived from the underlying bedrock formations (see figure 3). Where sandstone formations occur, the soils are mainly A 2-4 and A 4, sandy soils having very little coarse material, low plasticity and high capillarity, but they contain enough clay to have low permeability. Where the shale is exposed, A-7-6 soils with group indices in the range from 15 to 20 are most common; these are high-swelling, silty, bentonitic shales which have caused many of the construction problems in eastern Montana, including those at Fort Peck dam and the Grass Range to Malta highway connection. The geological descriptions are somewhat misleading when they imply that

there is a clear distinction between sandstone and shale formations. Actually, the sandstone formations contain numerous shale interbeds and the shale formations contain sandstone beds. Thus, the alluvium along valleys and swales is usually a combination of the above soil types with the deleterious features of the shale predominating.

CLIMATE

Rainfall in the area averages about 12 inches per year; about half of this occurs in the fall and winter as snow; late spring and early summer are usually the wettest seasons. Some of the materials are conducive to frost action, but moisture conditions are seldom serious enough to result in major damage to highways.

TOPOGRAPHY

The area studied is a part of the eastern Montana plains. The dominant topographic features are the major stream valleys and the moderately dissected plains between them. There are no mountains or highland features. Glaciation, which covered the northern part of the area, left only minor topographical features. Because of the soft underlying rocks, stream valleys tend to be shallow, wide and alluvium filled. The hard beds form exaggerated topographic features--cuestas, mesas, divergent drainages. Evidence of wind erosion, such as blowouts, is not uncommon. Examples of badlands topography have developed in several sandstone areas.

CONSTRUCTION MATERIALS

The location of various construction materials and the classification in which the materials in the area are placed was a major part of the problem under study.

Gravel associated with and derived from glacio-fluvial activity

is found along the Musselshell River in the northwest, along Dry Creek northeast of Van Norman, along Fort Peck Reservoir, and along the Redwater River in the east. In addition, good quality gravel occurs along the Musselshell River in the southwest and the Yellowstone River south of the area. The glacial and fluvial gravels are composed of predominantly hard, durable rock types derived from mountainous areas to the west and Canadian Shield areas to the north.

Gravel also occurs along many of the small perennial streams of the area. This gravel is a concentration of the harder portions of the sandstone and shale and will be referred to as local gravel in this report. The deposits tend to be thin, discontinuous, and poorly graded; furthermore, the gravel is often limited vertically by beds with high contents of expansive clay. Some gravel beds contain enough clay to create problems with plasticity and swell. Few pieces are very hard and loss in the Los Angeles wear test is usually high--commonly above the specification limits. Despite short distance of transport, individual fragments are usually rounded. The combination of some or all of these factors--particle shape, softness, poor gradation, and detrimental fines--usually results in a mediocre product; the thin erratic deposits, often under considerable overburden, result in a very high-cost product. Previous work indicates that hydrated lime, or portland cement modification, may offset some of the more detrimental physical aspects, but no method of accomplishing a drastic reduction in production costs is apparent.

Quarry sites in hard sandstone and baked shale provide one of the sources of materials at the present time. Only limited use has been made of the harder sandstone beds or "cap rock" as quarry material; this material is usually not easily available in sufficient quantities and its

durability when exposed to abrasion is poor. Where attempts have been made to use crushed sandstone as gravel surfacing, the results have not been satisfactory.

Some of the harder igneous dike material from Smoky Butte was used as rip rap at a dam near Jordan; the rock has proven durable.

Baked shale deposits are erratically distributed in the northern and eastern parts of the area. Although individual deposits are small, there is often a series of deposits in close proximity. Where the baked shale has been used as gravel surfacing, the results have been fair. This material is light and porous, and is not strong enough to resist severe traffic abrasion; the fines are usually non-plastic and have high absorptive capacity. This absorptive capacity presents a serious problem in producing an asphalt mat from baked shale. Asphalt contents from 10 to 13 percent are often required for a mix with long term stability, but absorption is relatively slow, and a rich mix cannot be placed or compacted.

The other road material which has been widely used is decomposed sandstone. Most of the testing thus far has been concentrated on this material. The range in quality of the sand is shown by test results in figure 4. The sand is composed of angular to moderately rounded quartz grains (75 to 90 percent), altered feldspar (0 to 5 percent), mafic and opaque heavy minerals (3 to 5 percent), and calcium carbonate (3 to 15 percent), and variable amounts of clay minerals. The type, grain size, and amount of clay minerals determine plasticity and swell characteristics, but size distribution, roundness, and composition are also factors determining suitability for highway use.

The sand has been used in several instances as surfacing on low traffic count roads; results have varied from fair to good depending on the quality of the sand, the underlying material, and, especially, on moisture conditions.

Sand was used both as base and as sand-asphalt mat on about 70 miles of road in semi-experimental projects constructed during 1947 and 1948. Although most of this mileage was still in service in 1963, from a maintanance standpoint, at least, the experiments were not very successful. Failure patterns--extensive mat cracking followed by disintegration and shoving of the mat--were relatively consistent despite variations in asphalt types, mat thicknesses, and base thicknesses. The shoulderless, steep inslope design used on these projects resulted in general edge failure. The prevalence of intensive mat cracking and displacement indicated serious strength deficiencies in both base and mat.

TEST METHODS

Maximum use of standard test methods was deemed desirable, but some of the materials, especially the sand, failed to show consistent results and therefore part of the project was directed toward investigation of test methods which would be meaningful for these materials.

The Marshall Method is used for flexible pavement design in Montana. Pavement experimentation was done by this procedure, but reproducibility of test results with sand-asphalt combinations was not satisfactory; much closer control of such variables as temperature and mixing conditions is required with these materials than with gravel.

For soil cement stabilization experiments, Portland Cement Association methods were used satisfactorily. Strength tests were run on 2-inch diameter by 2-inch high specimens; freeze-thaw tests were run on standard 4-inch by 4.6-inch specimens.

For asphalt stabilization experiments, there is no agreement on test methods. As a general comparison, a simple unconfined compression test was run using 2-inch diameter by 2-inch high specimens, a 1 square inch test plunger, and uniform rate of stress increase. One specimen was broken after three days curing at room temperature; duplicate specimens were broken after cure and three day immersion. Reproducibility of results was poor, but the test permitted rough classification of the materials.

The features that were desired in a test for asphalt stabilized base were:

1. Test be conducted about 77°F
2. Use of relatively large specimens (at least 4-inch diameter)
3. Specimens to be broken while subjected to confining pressure
4. Determination of moisture resistance

After considerable experimentation, it appeared that the Hveem stabilometer procedure offered the best possibilities with the equipment available. Later work indicated that cohesiometer values would be significant and a necessary adjunct to the "R" or "S" values. Moisture susceptibility was investigated by placing stabilometer specimens (in plastic molds with open ends) on saturated felt pads in the moisture room (100 percent humidity, 70°F) for various periods of time up to 28 days, and also by immersion of duplicate specimens in the molds.

Standard Proctor Method (AASHO T-99) was used for maximum density-

optimum moisture determinations as this is the test on which compaction specifications in Montana are based. In addition, each Proctor specimen was trimmed to 2½ inch height and tested in the stabilometer for "R" value.

Grading and liquid limit-plastic index were run by standard ASTM methods. In addition, liquid limit and plastic index were determined on the fraction passing the 200 mesh screen. Volume swell specimens molded according to standard Montana procedures were prepared for each sample. The volume swell procedure involves mixing 100 grams of the fraction passing the 10 mesh screen with enough asphalt to get good coverage, molding 2-inch diameter by 2-inch high specimens at a single plunger static load of 200 pounds per square inch, immersing the specimens for 8 days, and determining the amount of swell by mercury displacement before and after immersion. On many samples such additional tests as Field Moisture Equivalent, Linear Shrinkage, and Specific Gravity were also run.

BASE COURSE INVESTIGATIONS

GRAVEL

Work on gravel materials in the area was restricted to inventory, field examination of several deposits, and sampling these deposits. Considerable data is available on these materials from past use in various projects. Test results on typical gravels previously tested are shown in figure 6. (A) is glacial gravel from the area northeast of Van Norman, (B) is stream-concentrated material from Dry Creek near Jordan, (C) is colluvial sandstone gravel from the southern part of the area near Ingomar.

QUARRY MATERIALS

No intensive work was done on quarry materials for base course. Harder sandstone beds have many of the problems of the local gravel deposits; they are soft, quite variable within a deposit, and often thin, discontinuous, and deeply covered.

Scoria has been used principally for gravel surfacing and its use will be discussed under surfacing. Igneous material from Smoky Butte was used in a dam near Jordan, but the nature of the deposit must have required careful selectivity to get uniformly satisfactory material.

SAND

Because of its very wide distribution in the area, sand--decomposed soft sandstone--was the object of the bulk of testing. As previously mentioned, the sand has been used for both base course and sand-asphalt mat with fair to poor results. Several other states, however, have used sand extensively with reportedly good results.

Figures 7A and 7B show the results of "R" value tests on Proctor density specimens for some of the better sands. The maximum "R" values are barely adequate for base under low traffic secondary roads. The "R" values for most samples show sharp decreases for moisture contents greater than optimum (the point of maximum stability is commonly 1 to 3 percent drier than optimum moisture). Sample 35 is essentially cohesionless and shows nearly constant "R" value beyond optimum moisture.

In all cases it appears that stabilization is needed before the sands can be used as base for medium traffic primary and secondary roads. Experimentation was conducted with asphalt and portland cement stabilization. Selection of samples for this experimentation

was based on the test results shown in figure 5, geological source, and geographic source. Samples marked with an asterisk were selected for detailed study.

PORTLAND CEMENT STABILIZATION

The results of portland cement stabilization of six of the sand samples are shown in figures 8A, 8B and 8C. Minimum 7-day strengths based on Portland Cement Association data range from 240 psi to 280 psi; these strengths were obtained at cement contents ranging from 5 to 7 percent for the decomposed sandstone samples and at 9 percent for sample 35, the clean stream sand. Freeze-thaw resistance--the major criteria in soil cement design--was only satisfactory at cement contents of 6.7 to 9.0 percent, well above cement contents at minimum 7-day strengths, but below contents often required with soils in this class.

Although further testing of a specific deposit would be necessary, it appears that portland cement stabilization may be economically feasible in parts of the area where gravel is absent and sand can be produced with a minimun of treatment.

SAND-ASPHALT STABILIZATION

Sand-asphalt stabilization experiments drew much of the research effort on this project for several reasons:

1. A considerable amount of work, including construction, was previously done on sand-asphalt combinations and the result of this work can be evaluated to some extent.
2. Sand-asphalt surfacing and sand-asphalt base stabilization have many of the same problems.

3. Asphalt stabilized sand should have less tendency to crack because it does not have the volume change or the hydration reactions of portland cement.
4. Sand-asphalt stabilization should be amenable to stage construction.

Results of a simple immersion-compression test on all of the specimens are shown in figure 9. Further work on the samples selected for additional testing showed that the strength results from this test cannot be reproduced within acceptable limits. These tests do permit some pertinent general observations, however. It is apparent that the usual aggregate specification limits on volume swell (max. 15 percent), liquid limit (25), plastic index (6), are also applicable to sands; there is a fair correlation between such tests as field moisture equivalent and geological formation. It is also apparent that even where there is a strong similarity between sand samples, the test results can vary widely.

Typical grain-size analyses for sand samples from several formations are shown in figure 10A, 10B and 10C; the concentration of particles in a small range of sizes is evident from both the "percent passing" curve and the histogram. Although there may be some formational peculiarities in the grading, there is a gross similarity between the sand samples from all of the formations. This similarity explains the initial concentration of the work on the sands; procedures which are effective with any of the sand samples would lead to solutions for sands throughout the area. The poor size distribution which is a characteristic of these sands, explains the difficulty in

reproducing results with "standard" procedures. Instead of one or two major factors such as grain interlock, there are numerous factors which assume major importance under certain conditions and some of these--grain shape and surface for instance--are not evaluated by our standard tests.

After investigating various test procedures commonly used with sand mixes, it appeared that the Hveem stabilometer or triaxial shear procedures offered the most comprehensive and usable results. A search of the literature indicated that asphalt contents for initial stabilization tests are generally determined from the empirical data--percent of sample passing the 200 mesh screen or a grading formula such as: Percent asphalt = 0.015 (percent + 10M) + 0.02 (-10M + 40M) + 0.03 (-40M + 200M) + 0.09 (-200M). The results of Hveem stabilometer tests using SS-1H emulsions at these arbitrary asphalt contents, are shown in figure 9. These samples were mixed with the minimum amount of water which would permit a uniform mix (usually about the optimum moisture of the soil), then the indicated amount of SS-1H was incorporated; the sample was dried to optimum moisture (water + emulsion) and compacted in a 4-inch ID plastic mold. The sample in the mold was cured until there was approximately 4 percent water remaining (an arbitrary estimate of field drying efficiency). One sample was tested for "R" value in the stabilometer and duplicate samples were put on felt pads in the moist room for various periods of time and then tested.

The "R" values are related to the moisture and also to the density, so close control of these factors is necessary. Specimens which were tested after complete immersion showed much greater absorption and greater strength loss than specimens which were placed on felt pads;

in practice, a short term moisture susceptibility test would be more desirable. There were large differences in cohesion between the various samples, which is not adequately shown in the test results; cohesiometer values should be obtained in addition to the "R" values.

The SS-1 emulsion was selected for use after a series of preliminary tests indicated that cationic emulsions, even the slow setting types, "broke" almost immediately on contact with the sand. The minimum moisture content required for uniform mixes was generally about the optimum moisture for the sand.

Other work done by the laboratory indicated that adhesion of asphalt to mineral surfaces can be greatly improved by additives. To determine if the benefits from additives would be reflected in improved water resistance, eleven additives in various proportions from ½ to 1 percent (by weight of 100-120 asphalt cement) were hot mixed with sample 32; although several additives may warrant additional investigation, neither the dry nor the immersed strengths were significantly better than the strength of specimens without additive.

SURFACING COURSES

Previous attempts to overcome the poor distribution of quality surfacing materials have centered on sand-asphalt combinations and baked shale surfacing; in addition, maintenance has made wide use of stream gravel derived from sandstone and shale. The sand-asphalt combinations required extensive maintenance 2 to 3 years after construction regardless of the sand or asphalt used. The baked shale and local gravel have given fair to good service. Crushed sandstone surfacing has only been used a few times; the results were fair to poor.

Marshall tests results on sand-asphalt combinations are shown in figure 11. The most persistent features of these tests are the high voids of the mixes and their sensitivity to excess asphalt. The high voids may partly explain the strong tendency for these mats to crack. Not only is the strength of the mixes largely due to the asphalt, rather than to interparticle friction, but also, as shown in work by the Asphalt Institute, the tendency for the asphalt to harden is related to the amount of voids in the mix.

The relation of high voids to gradation was investigated by addition of portland cement to sample 16 and by blending sample 16 with various proportions of -200 mesh from local alluvial silt (sample 5). The results of these tests are tabulated below; the resulting grading curves for the mixtures are shown in figure 12. Although the tests, run in three series, emphasize the problem of obtaining consistent results, they show in general that a beneficial reduction in voids of these asphalt mixes requires addition of relatively large amounts of filler. The cost of commercial additives would appear prohibitive; addition of natural fines may be practical in some areas, but problems arise involving high asphalt contents and there is difficulty achieving a consistent, uniform mix.

The decomposed sandstone samples contained enough clay to impart some cohesive strength. The alluvial sand is virtually cohesionless and all specimens made with this sand were fragile and weak; blends of this sand and several silty samples reduced the voids and resulted in Marshall strengths near 1500 pounds as compared to strengths less than 500 pounds from sample 35 alone.

Addition of coarse material should likewise cause significant improvement in voids and strength of the sand mixes, but the absence of

good coarse material is the crux of the problem.

Sample 16

% Filler	% Asphalt	% Voids	Marshall Strength
0	6.5	24.8	1904
4 PC	6.5	25.8	1020
6 PC	6.5	25.4	1094
8 PC	6.5	24.5	1143
10 F	7.0	22.0	1579
12 F	7.0	21.6	1648
14 F	7.0	20.3	2062
20 F	7.5	18.7	1535
25 F	7.5	18.3	1700

PC is portland cement as added %
F is -200M alluvial silt (sample 5)

BAKED SHALE SURFACING

Baked shale has been widely used in the area as surfacing gravel, but it is seldom used in bituminous pavements because of its very high absorption. Furthermore, the rate of absorption is relatively slow so that large amounts of asphalt cannot be added to an initial mix. Although the strength of baked shale is minimal by some standards, tests indicate that solution of this absorption problem might result in a high-strength scoria-asphalt mix.

Filling the baked shale pores before adding asphalt is one way of reducing absorption. Sodium silicate-calcium chloride treatment appeared to do this to some extent, but the procedure is involved and expensive.

Solution of the absorption problem (but not the high asphalt requirement) appears possible by addition of part of the asphalt, followed by a warm weather cure and absorption period, the addition of the remainder of the required asphalt, and placing the mix. This procedure presents some serious problems in control of the asphalt content and thoroughness of absorption.

On the assumption that most of the absorption is in the fines, a combination of +4 mesh baked shale with local sand fines were tested.

Marshall test results are shown in figure 13. A curing period greatly added to the strength of the mixes when they were compacted well above the optimum asphalt content as indicated by the Marshall test. The gap in grading between the sand and the baked shale was shown by extractions to be partly filled by the breakdown of the baked shale during impact compaction.

SUBGRADE

The possibility of modification of the more objectionable soils--the highly expansive A-7-6 soils--in the area was investigated in connection with another project. Test results of the clay-shale are shown in figure 14. Portland cement and hydrated lime were used individually as stabilizers; as work by several researchers has shown good results with phosphoric acid or combinations of hydrated lime and portland cement, these were also used. Variation in curing procedures and in the sequence of additives and water were investigated and the procedure yielding the best strengths was used. Test results with the shale from this project clearly show that even large amounts of additives do not satisfactorily modify the shale. Modification of the plasticity and swell properties of the shale was accomplished at high additive concentrations. Strength gains at higher additive concentrations were small, and at no point was the strength commensurate with the amount of additive. This work is based on Bearpaw shale subgrade samples from a short section between Ingomar and Vananda, but further work may show that the general conclusions apply to all of the highly expansive shales in the area.

Another aspect of subgrade materials, which received attention, involved factors determining the condition of unpaved roads. Most of the

selected sites were on unsurfaced roads, but one site had a thin sand surfacing. Samples were taken at adjacent good and poor sections of the road. Results of tests to determine to what extent materials, drainage and topography affected the roadway condition are shown in figure 15. Field observations indicated the importance of drainage and topography. The laboratory test results strongly indicate the importance of materials characteristics. Actually, there was a coincidence of controlling factors; topographic conditions resulting in poor drainage conditions also result in concentration in the soil of the finer and less desirable portions of the bedrock. Design factors have a minimum influence on the selected sites.

CONCLUSIONS

BASE COURSE

Results of the study indicate that stabilization is required for local sands before they can be used satisfactorily for base course. Tests on portland cement stabilization showed adequate strength and durability for selected samples in the range from 6 percent to 9 percent cement. Because of erratic test results, no definite conclusions could be reached with regard to the feasibility of asphalt stabilization.

The most practical use of sand is for subbase where the subgrade is expansive shale or clay. For this use sand can replace gravel, although the sand must be protected from erosion and extreme moisture changes. As subbase, the sand will adequately prevent the intrusion of clay into the base course gravel or minimize displacement of overlying stabilized base. The sand should have sufficient fines to bind it together and make it essentially impermeable. Very few of the sands in the area are too "clean" for good subbase, but there are quite a number which are un-

suitable because of high plastic indices and swell potential.

SURFACING

The high void volume of the sand-asphalt mixes is an inherent characteristic which adversely affects the use of these mixtures for surfacing. Addition of sufficient asphalt to reduce voids appreciably creates a very unstable mixture; addition of mineral fillers in amounts up to 10 percent does not result in significant improvement; addition of natural fines to partly rectify the grading deficiences complicates mixing procedures and increases asphalt requirements.

Baked shale-asphalt surfacing is only practical if absorption can be reduced to where asphalt contents are reasonable and mats have long term durability and stability. Experiments along this line were only partly successful.

Use of sandstone-derived stream gravels results in a mix of adequate stability, but use of hydrated lime or portland cement should be considered where swell tests indicate adverse fines characteristics.

DESIGN CONSIDERATIONS

Examination of previous test sections indicates that the most important element in geometric design of roads using sand base involves the shoulder area. These test sections used a base material of marginal maximum strength and a narrow-shoulder, steep-inslope design. In addition, the mats were from 1½" to 2" in thickness, which, by any design based on subgrade strength, is structurally unsound. The maximum strength of the sand is developed at moisture contents slightly below optimum and is partly dependent on confining pressure which is a minimum at the shoulders. There is a moderate to sharp decrease in strength at moisture

contents in excess of the moisture at maximum strength. Because of the high capillarity of the sand, it easily loses or acquires moisture at the critical shoulder areas. Certainly any design utilizing sand should include adequate shoulders and inslopes designed with allowance for adverse moisture conditions and reduced strength. Observations of cracking of rigid base sections in other areas (mostly over A-7-6 subgrades) indicates that such shoulder design considerations are equally pertinent to rigid bases.

For thickness design on stabilized base sections it has been the practice to use considerably thinner total sections that would be indicated by exudation-expansion pressure tests or Allen classification criteria. Cracking is common to such rigid base sections, and the unevenness associated with this cracking where subgrade is especially expansive would indicate that attention should be given to increasing the total thickness by addition of non-expansive subbase where adverse subgrade conditions are encountered.

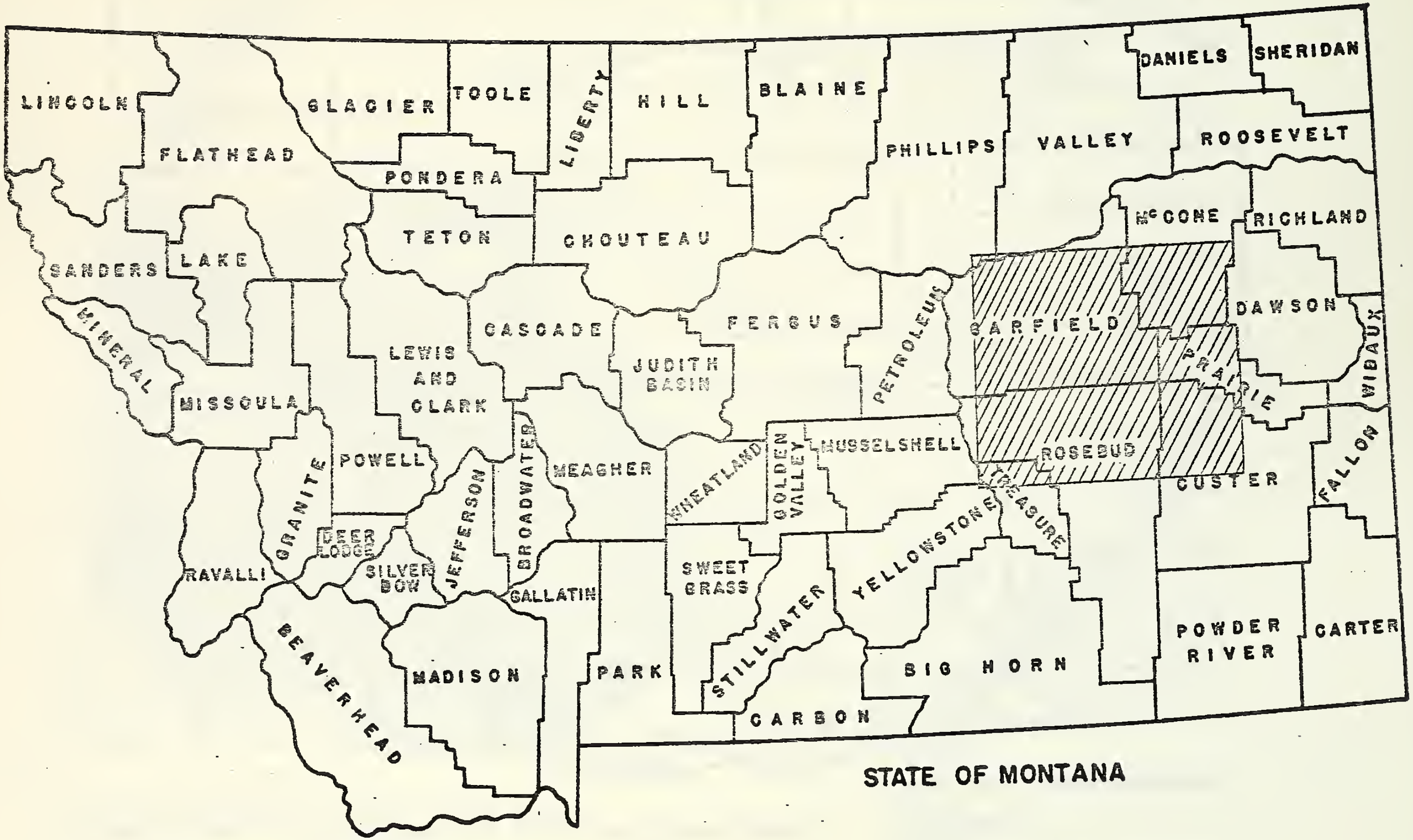


FIGURE #1 INDEX MAP OF SAMPLING AREA

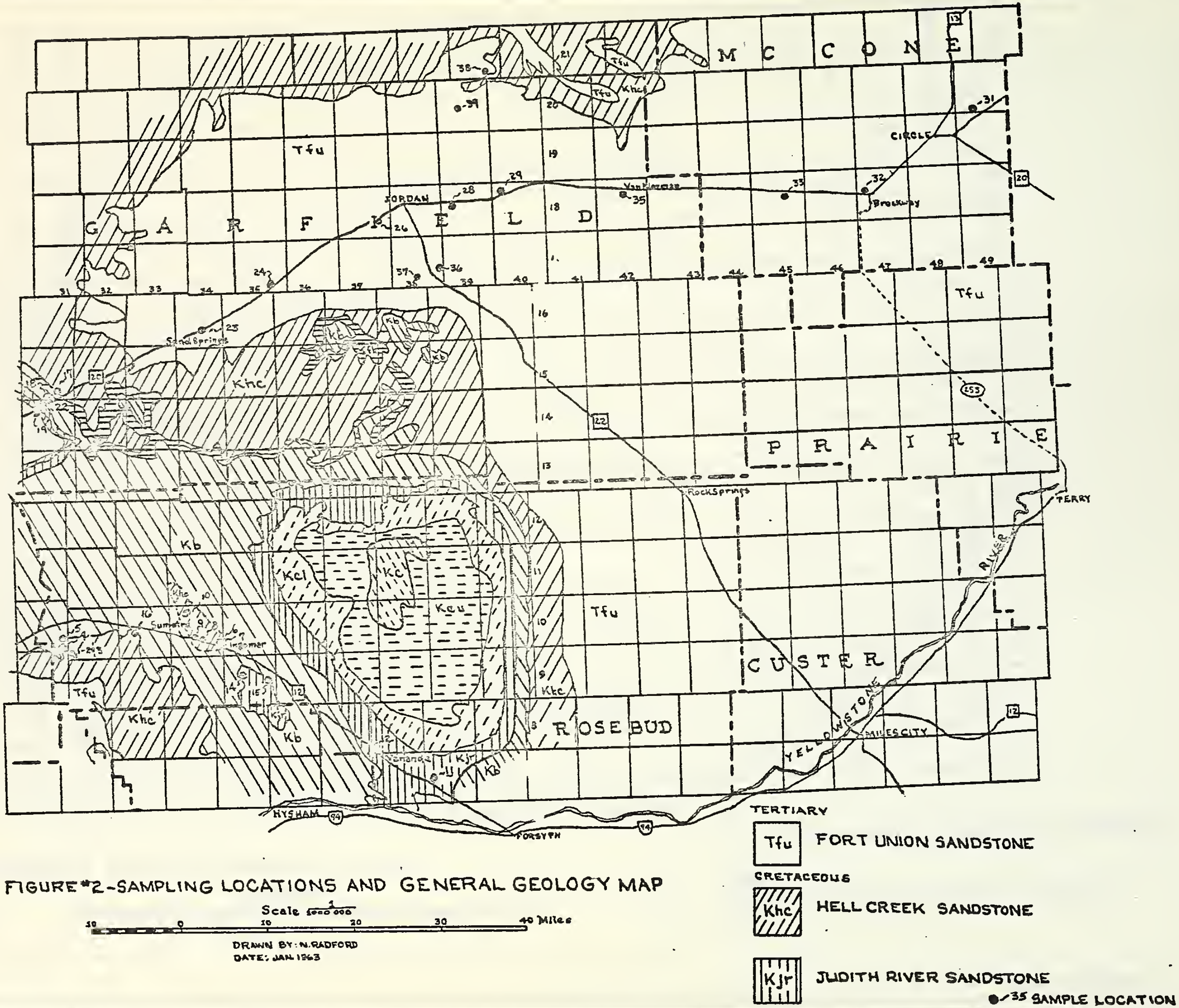


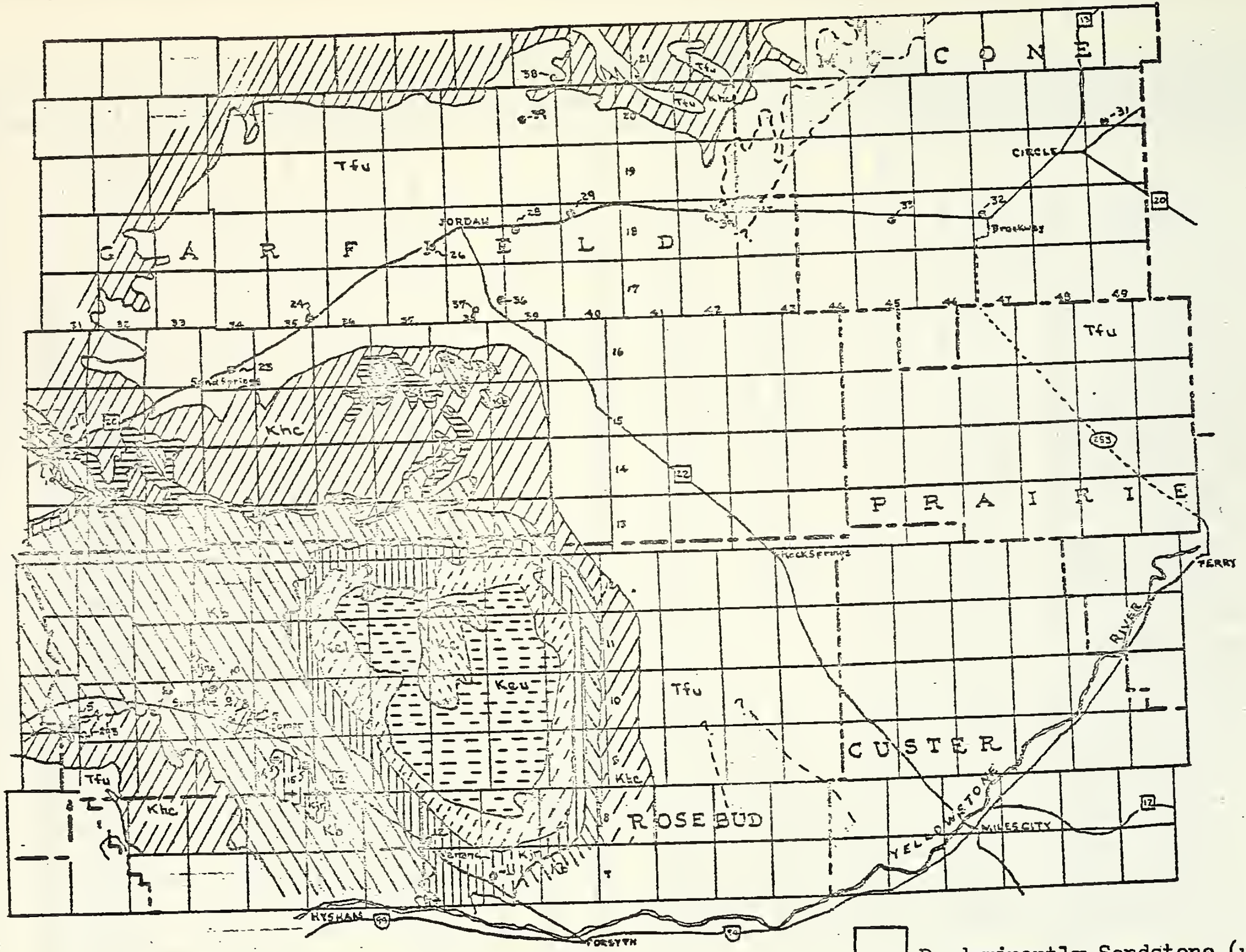
FIGURE #2-SAMPLING LOCATIONS AND GENERAL GEOLOGY MAP

Scale $\frac{1}{5000000}$

10 0 10 20 30 40 Miles

DRAWN BY: N. RADFORD

DATE: JAN. 1963



Predominantly Sandstone (unshaded)

Predominantly Shale (shaded)

FIGURE 3 General Lithography of Area

DRAWN BY: N. RADFORD
DATE: JAN-1963

SAMPLE CHARACTERISTICS

Sample	Location Sec,	T,	R	Soil Class	Percent Passing 4M	10M	40M	200M	270M	-40M LL	PI	-200M LL	PI	FME	Linear Shrink	[1]Optimum Moisture	Maximum Density	Specific Gravity	Volume Swell
1	31	10N	32E	A 3	83	81	76	9	6	NP	NP	34	4	27%	0.0%	------	------	------	3.9%
2	31	10N	32E	A 2-4	75	71	61	11	7	NP	NP	NP	NP	26	0.0	------	------	------	3.5
3	31	10N	32E	A 4(8)	100	97	92	85	83	33	8	36	10	31	7.2	------	------	------	31.0
4	36	10N	31E	A 4(1)	100	100	99	41	25	20	NP	24	3	23	1.9	13.7%	114.0 #/ft^3	2.64	11.0
5	25	10N	31E	A 4(5)	100	100	100	60	51	21	2	27	3	22	2.2	------	------	------	12.0
6B	25	10N	34E	A 2-4	100	100	99	27	20	24	NP	33	8	24	1.3	------	------	------	9.4
7	25	10N	34E	A 4(2)	100	97	92	44	35	24	NP	24	NP	26	0.0	------	------	------	12.0
8	27	10N	34E	A 2-4	99	98	93	20	11	NP	NP	28	NP	23	0.0	13.8	109.3	2.64	9.6
9A	28	10N	34E	A 2-4	95	95	92	15	10	NP	NP	30	4	27	0.0	------	------	------	5.1
9B	28	10N	34E	A 6(2)	99	95	75	41	33	35	14	39	17	28	4.9	------	------	------	17.0
10	5	10N	34E	A 2-4	100	100	96	23	19	22	NP	38	10	23	1.9	15.2*	110.6*	------	12.0
11A	19	7N	39E	A 2-4	99	95	88	12	10	NP	NP	44	10	27	0.0	------	------	------	6.7
11B	19	7N	39E	A 4(2)	100	100	100	47	39	32	7	44	21	27	3.1	15.8*	108.5*	------	9.6
12	31	8N	38E	A 2-4	100	99	98	35	29	30	4	42	8	28	3.6	17.9*	107.4*	------	24.0
14B	9	9N	35E	A 2-4	93	92	89	18	15	NP	NP	29	5	19	0.6	12.8	111.0	2.64	5.3
15	24	9N	35E	A 3	99	98	90	8	6	NP	NP	37	6	28	0.0	------	------	------	4.4
16	28	10N	33E	A 2-4	100	100	99	15	10	NP	NP	31	4	25	0.4	14.2	108.8	2.66	5.7
17	35	15N	31E	A 2-4	100	100	98	16	10	NP	NP	36	7	26	0.4	15.6	108.7	2.68	7.5
18	34	15N	31E	A 2-4	100	99	97	14	8	NP	NP	34	6	27	0.0	------	------	------	5.4
19	4	14N	31E	A 2-4	100	100	84	18	15	34	NP	81	50	30	1.2	------	------	------	30.0+
22	4	14N	31E	A 2-4	100	97	82	18	11	NP	NP	33	5	29	0.0	------	------	------	2.2
23	22	16N	34E	A 2-4	100	100	93	14	10	36	5	70	9	37	1.6	------	------	------	30.0+
24	34	17N	35E	A 2-4	100	100	100	21	13	22	NP	32	3	28	0.3	------	------	2.72	6.3
26	26	18N	37E	A 2-4	74	70	52	10	8	26	NP	52	20	29	1.6	------	------	------	19.0
28	8	18N	39E	A 2-4	100	99	93	13	10	NP	NP	56	4	26	0.6	------	------	------	16.0
29	7	18N	40E	A 2-4	100	100	87	9	6	NP	NP	45	11	29	0.0	------	------	------	6.1
31	32	20N	49E	A 2-4	100	100	99	14	10	NP	NP	38	6	28	0.0	------	------	------	6.1
32	18	18N	47E	A 2-4	100	100	98	12	9	NP	NP	33	NP	28	0.0	15.1	105.0	2.69	3.2
33	15	18N	45E	A 2-4	100	100	100	30	16	NP	NP	NP	NP	27	0.6	------	------	2.72	2.8
35	10	18N	42E	A 2-4	100	100	97	8	3	NP	NP	NP	NP	27	0.0	13.9	97.7	2.70	1.6
36	24	17N	38E	A 2-4	100	100	97	22	16	NP	NP	32	10	27	0.6	------	------	------	5.8
37	34	17N	38E	A 4(7)	100	98	94	72	66	29	3	32	4	29	4.8	18.5*	105.8*	2.69	16.5
38	35	21N	39E	A 2-4	100	100	98	20	19	NP	NP	74	42			------	------	------	30.0+
39	21	20N	39E	A 2-4	100	100	92	17	11	NP	NP	35	3			------	------	------	4.9

(1) OM & Md by standard Proctor except (*) by Harvard Method

Figure 4

PRELIMINARY EVALUATION TESTS(1) FBV MACHINE

Sample	FBV with SC-3						FBV with SS-1						Remarks
	5% SC-3		Ret. Stab.	7% SC-3		Ret. Stab.	3% SS-1		Ret. Stab.	5% SS-1		Ret. Stab.	
	Dry	Wet		Dry	Wet		Dry	Wet		Dry	Wet		
	psi	psi	%	psi	psi	%	psi	psi	%	psi	psi	%	
1	45	26	58	45	35	78	300	98	33	292	97	33	Sandstone
2	70	36	51	63	34	53	386	105	27	403	137	34	Sandstone
3	410+	--	0	410+	--	0	---	---	--	---	---	--	Shale
4	80	30	38	82	32	39	---	---	--	---	---	--	Colluvium(road material)
5	162	23	14	106	33	31	600+	32	5-	600+	76	13-	Colluvium
6B	120	--	0	125	30	24	1030	72	7	950	144	15	Sandstone
7	95	26	24	80	38	48	---	---	--	---	---	--	Sandy Shale
*8	62	25	40	61	28	46	460	63	14	605	150	25	Sandstone
9A	69	25	36	58	30	52	---	---	--	---	---	--	Sandstone
9B	410+	--	0	410+	--	0	---	---	--	---	---	--	Sandy Shale
10	120	--	0	139	63	45	---	---	--	---	---	--	Colluvium (road)
11A	54	28	52	63	28	44	280	56	20	275	103	37	Sandstone
11B	235	--	0	248	--	0	---	---	--	---	---	--	Sandy Shale
12	410	--	0	370	--	0	---	---	--	---	---	--	Sandy Shale
*14B	81	35	43	39	--	90	710	37	9	395	71	18	Sandstone
15	35	30	86	43	25	58	350	103	29	188	165	88	Sandstone
*16	80	35	44	69	35	51	---	---	--	---	---	--	Shaley Sandstone
*17	110	45	41	91	70	77	---	---	--	---	---	--	Sandstone
18	68	60	88	77	38	49	425	97	23	500	193	39	Sandstone
19	72	--	0	65	--	0	---	---	--	---	---	--	Clayey Sandstone
22	35	19	54	53	20	38	406	59	15	445	73	16	Sandstone
23	56	8	14	92	7	8	900+	27	3-	639	72	11	Shaley Sandstone
24	33	12	36	41	16	39	850	80	9	672	136	20	Sandstone
26	103	14	14	75	19	25	741	64	9	647	67	10	Sandstone
28	50	11	22	47	12	26	683	74	11	529	144	27	Sandstone
29	20	15	75	23	15	65	211	84	40	262	135	52	Sandstone
31	24	18	75	22	20	91	221	56	25	264	82	31	Sandstone
*32	18	19	106	26	18	69	310	110	35	336	176	52	Sandstone
33	36	10	28	34	22	65	329	86	26	399	116	29	Fine Sandstone
*35	10	21	210	8	9	112	151	97	64	172	144	84	Stream Sand
36	42	25	60	53	25	47	850	97	11	713	181	25	Sandstone
37	145	12	8	163	--	0	880+	27	3-	776	43	55	Colluvium (road)

(1) All tests at room temperature (±75% F). Dry samples cured 2 days at room temperature.
Wet samples immersed for 2 days after curing.

* Samples selected for more detailed work.

Figure 5

	Percent Passing		
Size	Sample A	Sample B	Sample C
3"	97		100
2"	93	100	98
1"	89	98	88
3/4"	87	96	83
1/2"	81	93	73
3/8"	75	88	66
4M	55	72	52
10M	37	53	41
20M	26	37	36
30M	23	32	36
40M	21	28	35
50M	19	23	35
60M	17	21	34
80M	13	17	31
100M	11	15	26
200M	7	12	22
270M	6	11	17
Liquid Limit	20	U20	22
Plastic Index	0	NP	1
Volume Swell	9.3%	8.5%	21.0%
L. A. Wear Loss	36%	43%	58%
Unit Weight	124 pcf	113 pcf	

A is glacio-fluvial; B is local stream gravel; C is sandstone colluvium

Figure 6 Typical Gravel Test Results

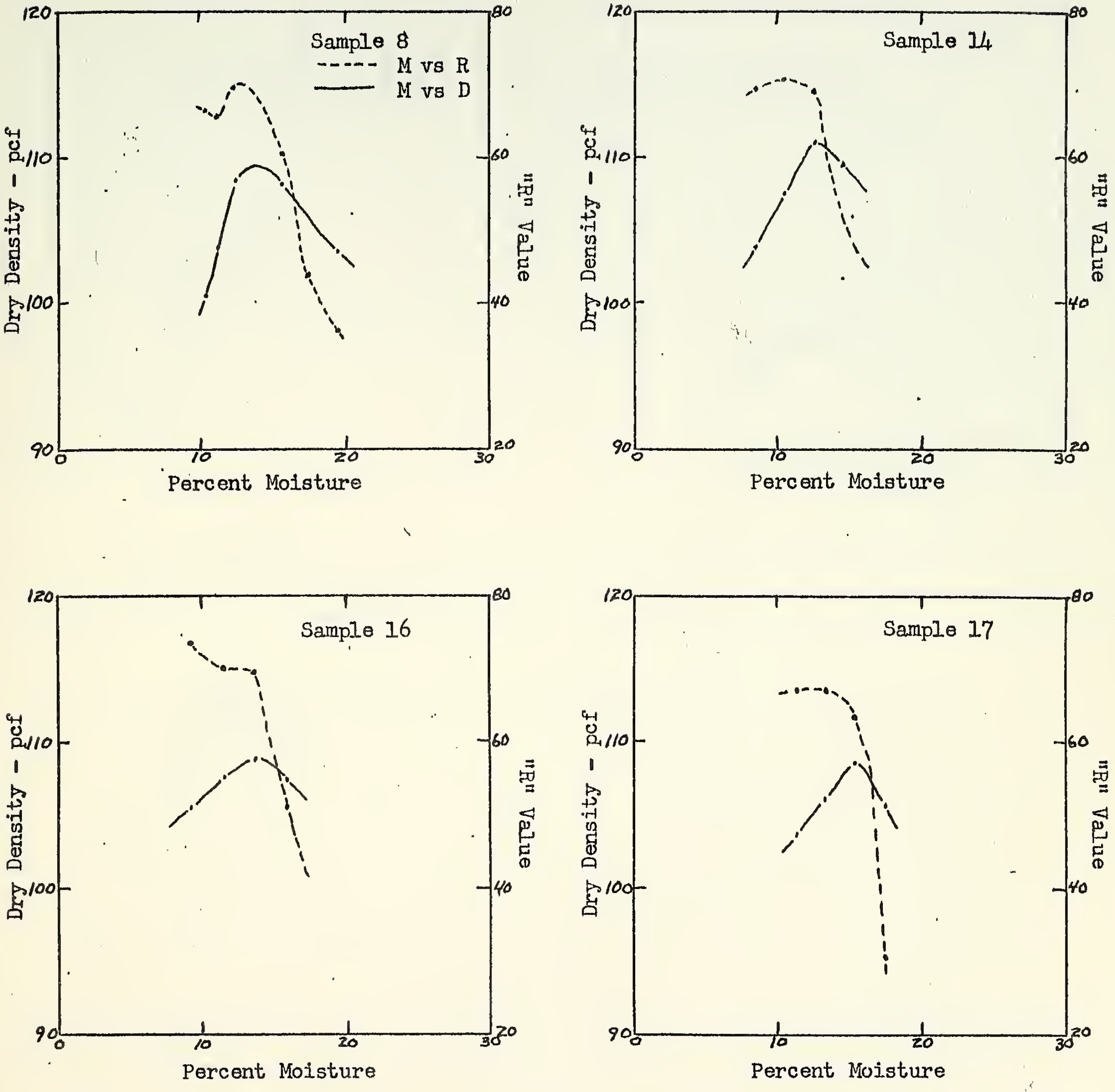


Figure 7A. Proctor Compaction Data vs Hveem Stabilometer "R" Values

[illegible]

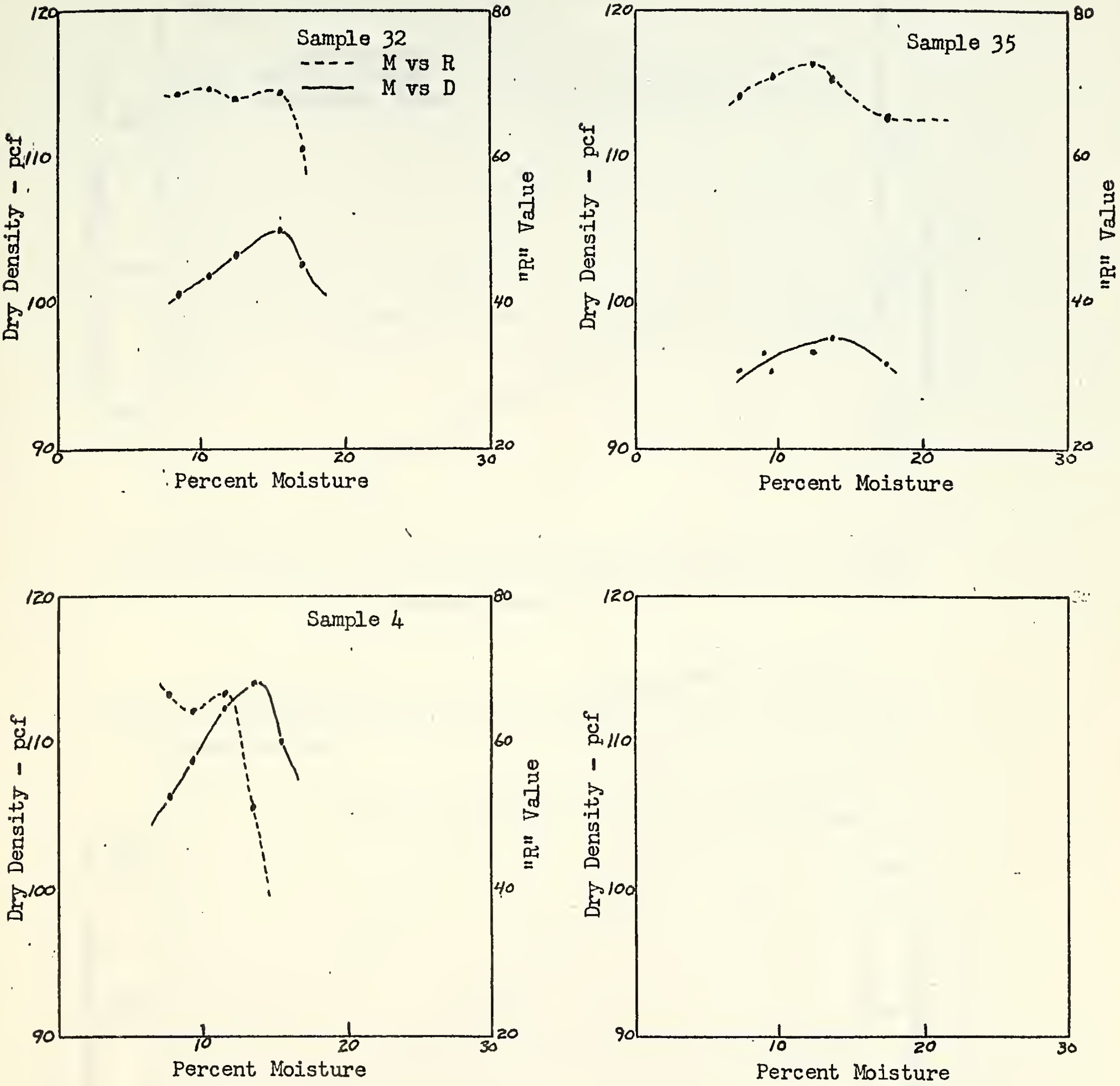


Figure 7B Proctor Compaction Data vs Hveem Stabilometer "R" Values

[illegible]

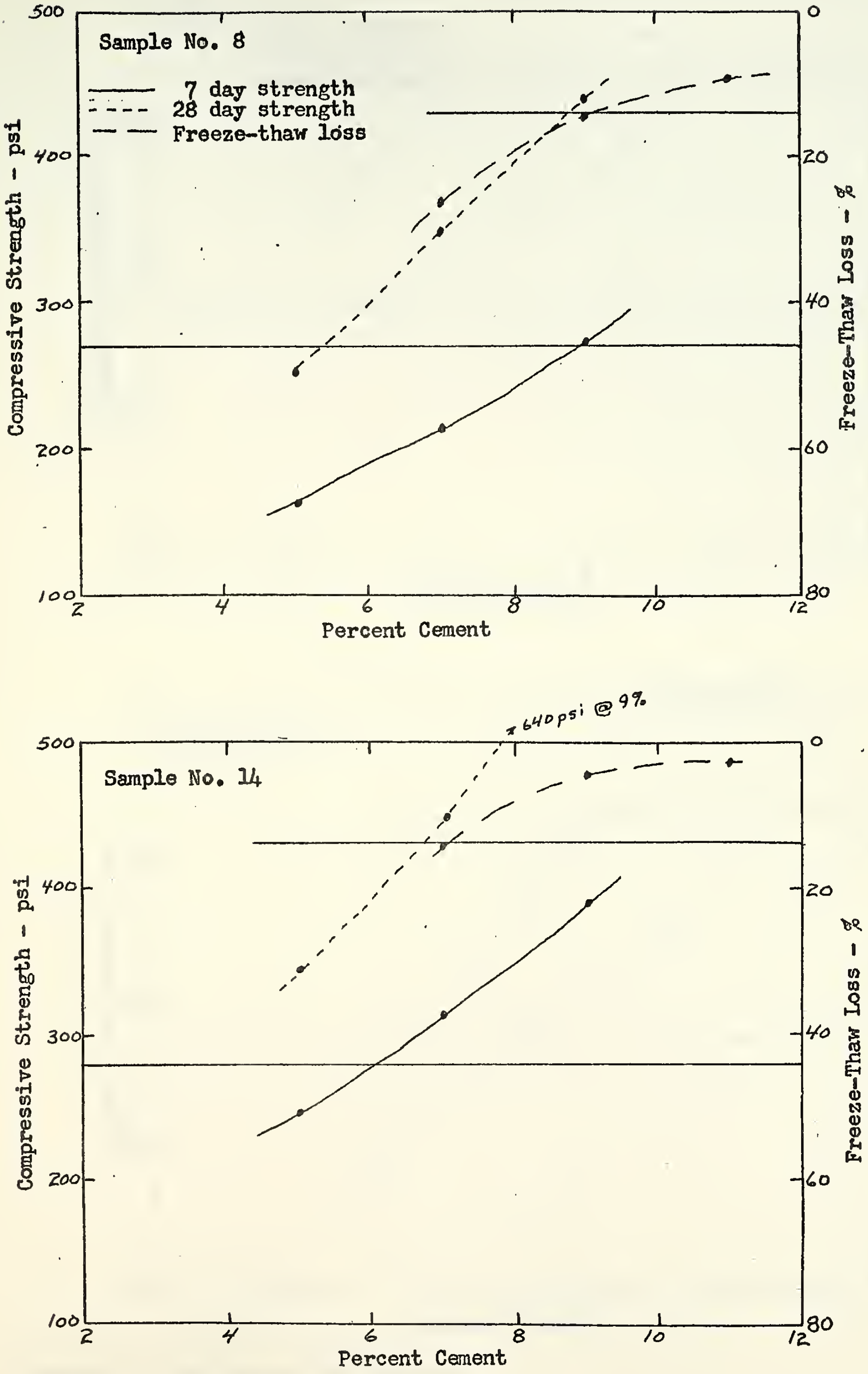


Figure 8A Soil Cement Test Results

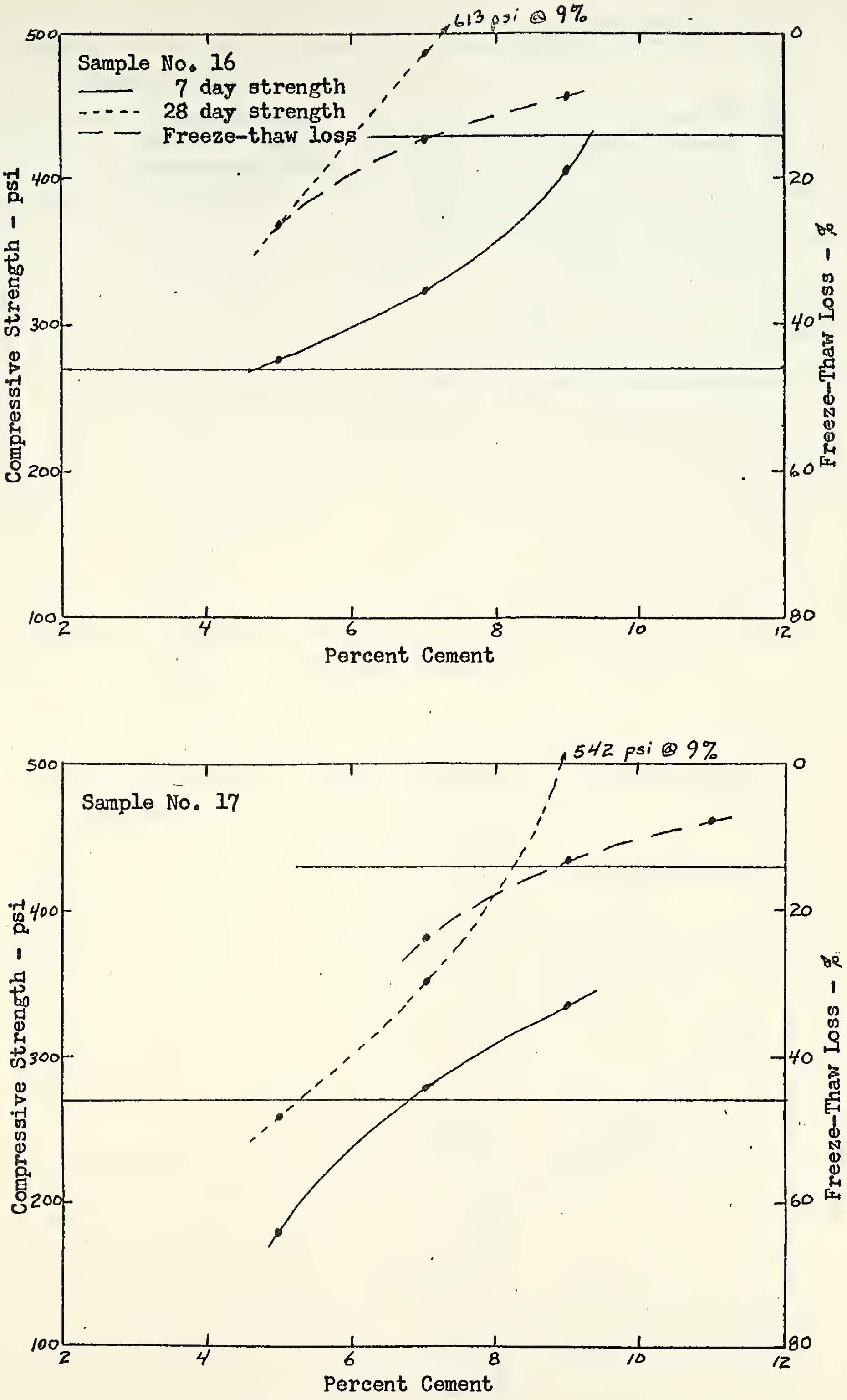


Figure 8B Soil Cement Test Results

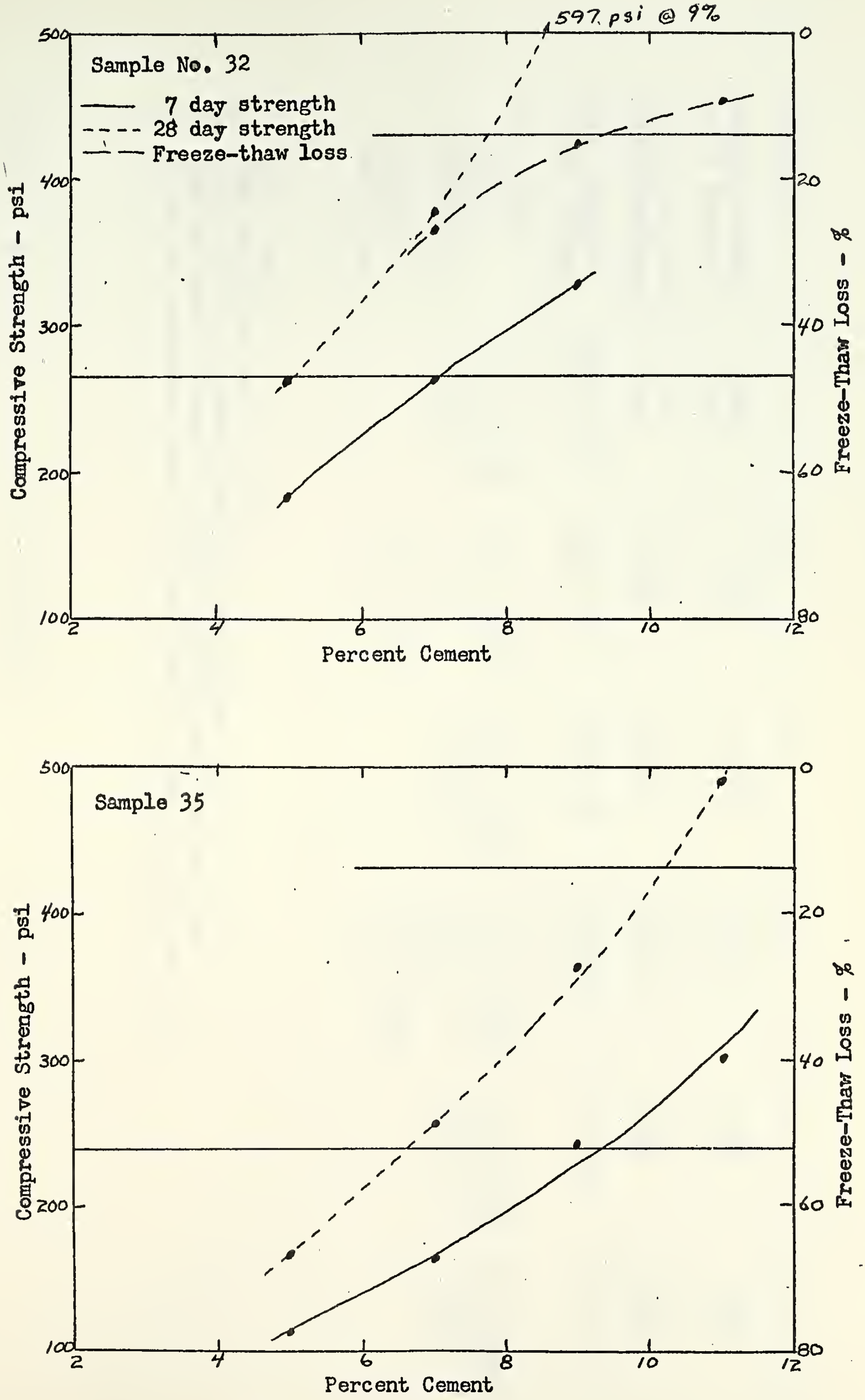


Figure 8C Soil Cement Test Results

Sample No.	8	14	16	17	32	35
Asphalt Content	4.0%	3.8%	3.7%	3.3%	3.5%	3.2%
SS-1H Content	6.2%	5.9%	5.7%	5.1%	5.4%	5.0%
Optimum Moisture[1]	13.8%	12.8%	14.2%	15.6%	15.1%	13.9%
Mixing Moisture[2]	14.5%	21.8%	14.2%	22.3%	20.0%	12.4%
Max. Density[1]	109.3 pcf	111.0 pcf	108.8 pcf	108.7 pcf	105.0 pcf	97.7 pcf
Actual Density[3]	107.4 pcf	108.9 pcf	111.1 pcf	112.5 pcf		
"R" Values[5]						
Cured	69.5 (4.4%)	83.0 (2.0%)	83.0 (4.4%)	87.5 (1.8%)	69.5 (4.2%)	75.0 (4.5%)
Vapor[4]	66.0 (6.3%)	70.0 (4.2%)	75.5 (5.4%)	68.0 (5.5%)	65.0 (5.9%)	71.5 (6.0%)
Absorption	1.9%	2.7%	1.0%	1.2%	1.7%	1.5%
Immersed		63.0 (6.8%)		63.5 (9.3%)		
Absorption		4.9%		4.2%		

1) soil alone
2) does not include water in SS-1H
3) water + SS-1H = optimum moisture of soil alone
4) samples exposed to vapor in plastic molds on saturated felt pads
5) figure in parenthesis shows moisture content at test; all tests at 77°F

Figure 9 Stabilometer Tests of SS-1H Emulsion

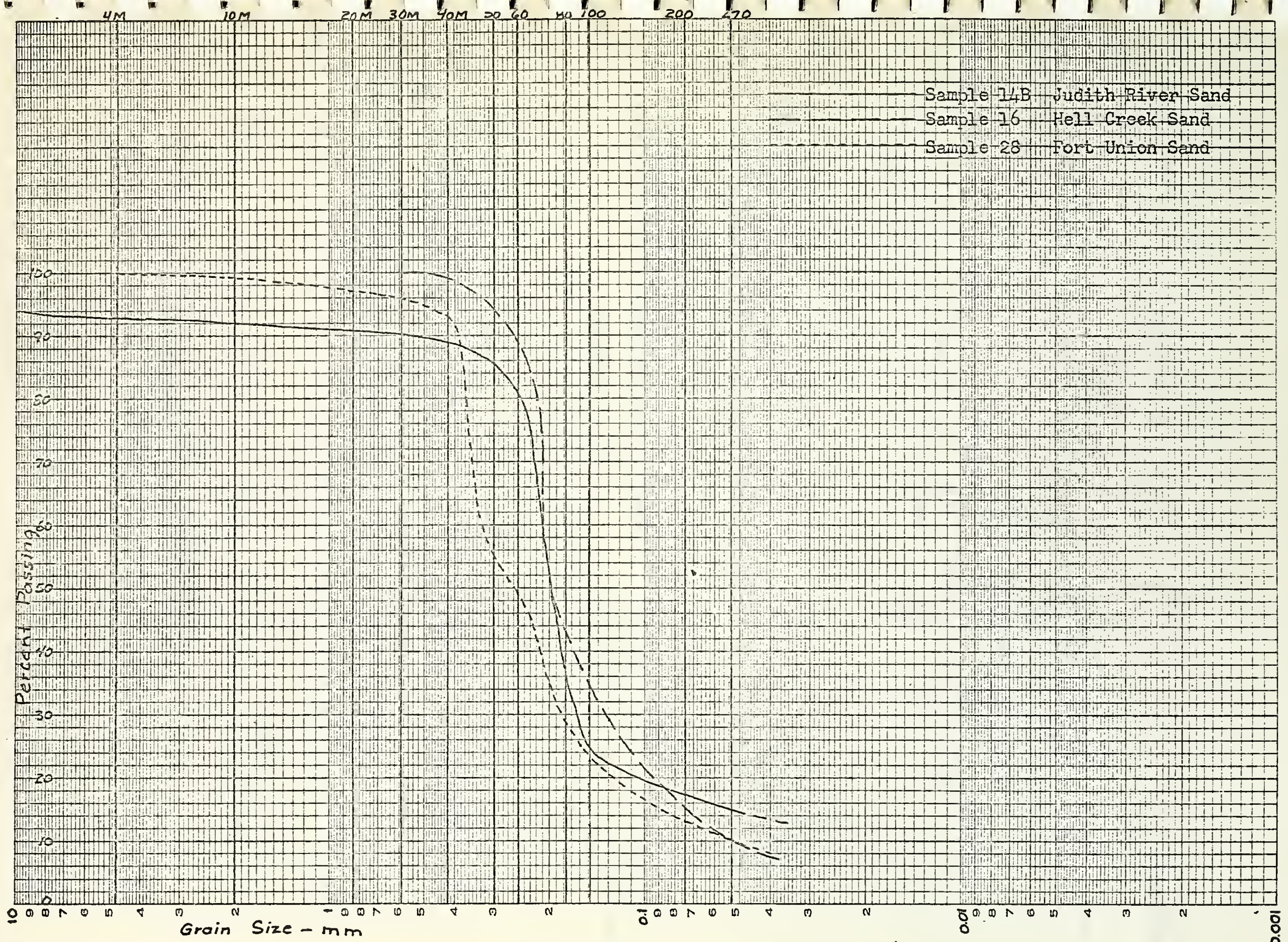


Figure 10

NO. 340-L410 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
4 CYCLES X 10 DIVISIONS PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.

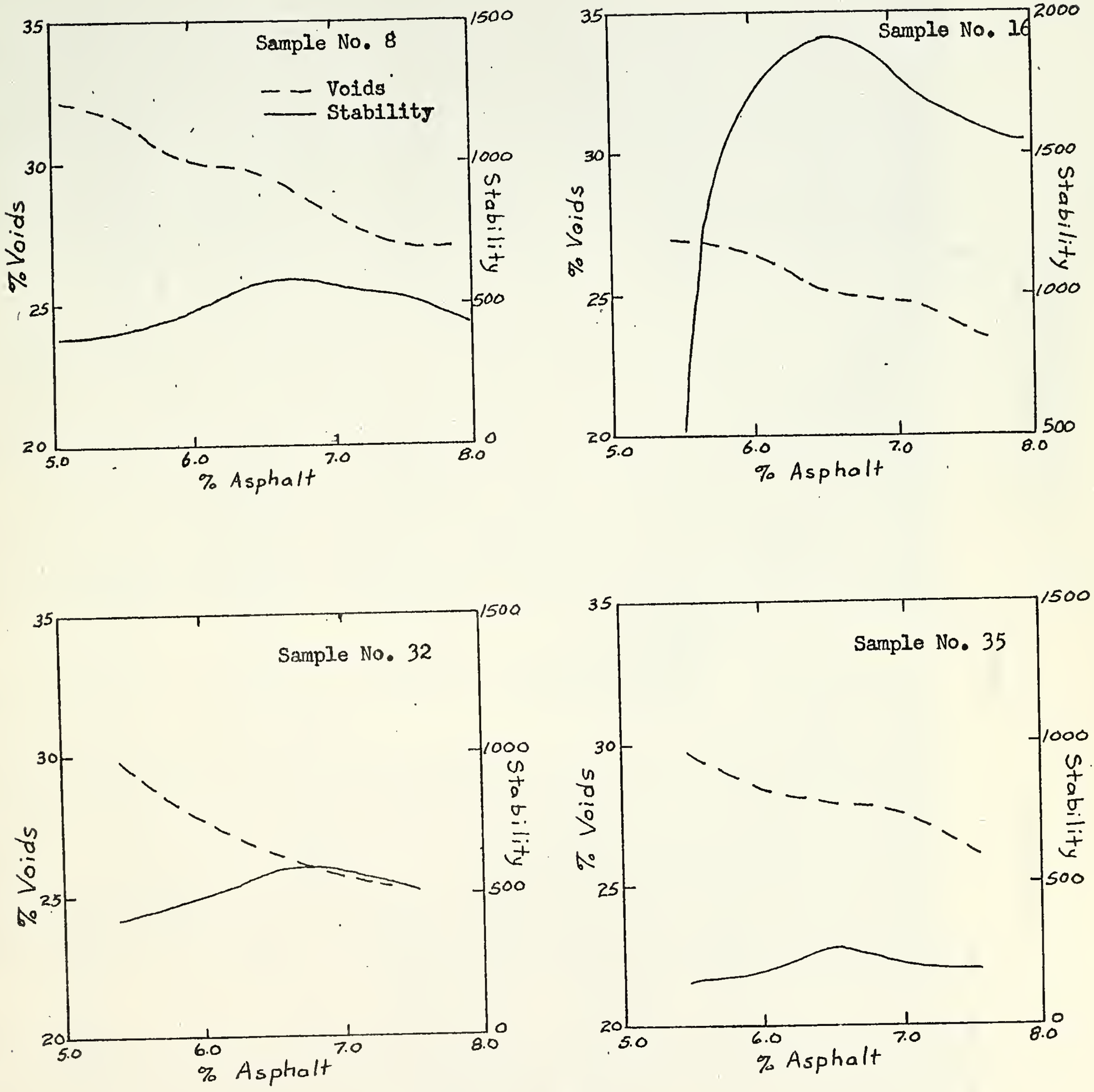


Figure 11 Marshall Test Results on Sand Samples
(Standard Method 100-120 Pen Asphalt)

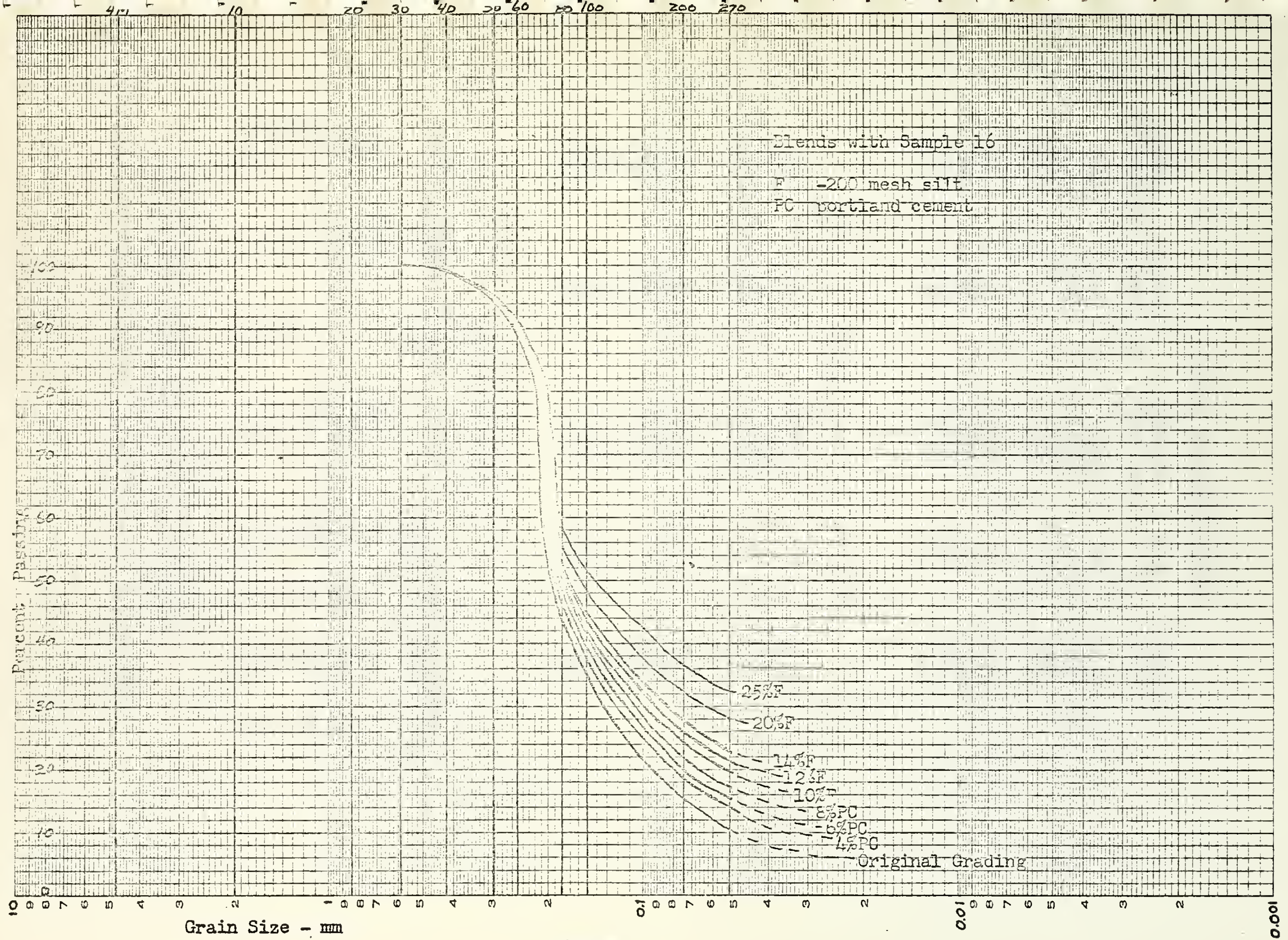


Figure 12

EUGENE DIETZGEN CO.
MADE IN U. S. A.

NO. 340-L410 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
4 CYCLES X 10 DIVISIONS PER INCH

Baked Shale (Sample 27)

Asphalt (100-120)	Percent Passing 4M	Marshall Stability	Flow	Percent Voids
7.0%	45	1108	7	17.1
7.3	50	1161	6	17.5
7.6	55	1406	8	18.0
7.9	60	1237	6	18.0
7.0	55	1540	8	18.3
8.0	55	1338	8	17.6
9.0	55	1540	11	16.4
10.0	55	1750	12	15.6

Blends of +4M baked shale and -4M sand

Asphalt (100-120)	Percent Passing 4M	Marshall Stability	Flow	Percent Voids
7.1	29	1710	9	12.3
7.5	40	2520	6	12.2
7.5	50	2000	8	13.4
7.5	60	2170	10	15.0
7.0	40	2547	8	12.6
7.5	40	1799	6	12.2
8.0	40	1958	6	11.8
8.5	40	2478	8	10.1
9.0	40	2737	9	9.7
10.0	40	2400	10	7.9
11.0	40	2800	13	7.5

Sample Gradings

Size	Sample 27	Sample 16	Blend[1]	Extraction Samples[2]			
				7.5%	9.0%	10.0%	11.0%
3/4"	100		100	100	100	100	100
1/2"	97		96	96	96	97	97
3/8"	86		83	85	84	87	85
4M	51		40	50	54	56	53
10M	34	100	40	41	43	43	42
40M	11	99	40	39	39	40	39
200M	4	15	6	25	10	20	10

1) 60% +4M from sample 27, 40% -4M from sample 16
2) 60-40 blend samples, percent asphalt indicated

Figure 13 Experiments with Baked Shale and Sand Blends

Sample	Additive	Gradation (percent passing) 10M	40M	200M	0.05mm	0.01	0.005	0.002	0.001	LL	PI	VS	Soil Class
1	None	100	96	92	89	70	60	48	34	65	23	30+	A 7-5(17)
2	3L*	100	93	80	76	57	46	34	--	55	10	30+	A 7-5(11)
3	6L	96	67	39	35	9	—	—	—	48	6	30+	A 5(1)
4	10L	92	54	34	31	5	--	—	—	43	6	31.1	A 2-5
5	6C	96	73	48	44	29	11	—	—	46	5	30+	A 5(3)
6	10C	92	62	37	34	21	—	—	—	44	6	32.2	A 5(1)

Sample	Additive	Strength 7 day
A	6C	84 psi
B	10C	101
C	14C	146
17	6C 4L	87
25	6C 3L	73
24	6C 2L	55
23	6C 1L	61
26	8C 1L	74
27	8C 2L	79
28	8C 3L	68
29	8C 4L	74

Sample	Additive	Strength 7 day cure	Strength Cure + 7 day Immersed
D	1% H_3PO_4	76 psi	7 psi
E	2% "	82	19
F	3% "	95	31
G	4% "	99	40
H	1% $CaCl_2$	30	Disintegrated
J	2% "	26	Disintegrated

* L - hydrated lime, C - portland cement, percent
All additives percent by weight of dry shale

Figure 14 Test Results of Bearpaw Shale Modification Attempts

Sample	Sand	Silt	Clay	Colloids	LL	PI	Volume Swell	Road Condition
1	21%	59%	20%	9%	24	2	4.0%	Good
2	36	26	38	25	33	13	30+	Poor
3	34	42	24	14	23	3	9.7	Good
4	48	24	28	20	35	13	30+	Poor
5	25	30	44	30	46	19	30+	Poor
6	21	35	44	29	41	17	30+	Poor
7	47	27	26	17	24	6	15.6	Good
8	31	39	32	16	31	9	15.6	Fair-Good
9	37	31	32	21	34	13	31.6	Poor
10	56	24	20	12	24	3	14.8	Good
11	35	30	35	20	30	11	30.4	Poor
12	57	25	18	8	20	3	10.9	Good

Average Colloids	Range Colloids	Average Clay	Range Clay	Average Silt	Range Silt	Average Sand	Range Sand
			Good Samples				
12.7%	8-17%	23.3%	18-32%	36%	24-59%	41%	21-57%
			Poor Samples				
24.2%	20-30%	36.7%	28-44%	29%	24-35%	34%	21-48%

Average LL	Range LL	Average PI	Range PI	Average Vol. Swell	Range Vol. Swell
			Good Samples		
24.3	20-31	4.3	2-9	11.8%	4-15.6%
			Poor Samples		
36.6	30-46	14.3	11-17	30+%	30.4-

Figure 15 Test Results of Road Samples